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**US Army Corps
of Engineers®**

Louisville District

Engineering Documentation Report

Green River Lock and Dam No. 3
Contract No. W912QR-08-D-0005
Task Order No. 0008
Rochester, Kentucky

**Stantec Consulting Services Inc.
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Prepared for:
USACE Louisville District
Louisville, Kentucky

May 18, 2011



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Mr. Jeff Esterle, PE, PG
USACE Louisville District
600 Dr. Martin Luther King Place
Louisville, Kentucky 40210-0059

Re: Engineering Documentation Report
Green River Lock and Dam No. 3
Contract No. W912QR-08-D-0005
Task Order No. 0008
Rochester, Kentucky

Dear Mr. Esterle:

Stantec Consulting Services Inc. (Stantec) is pleased to present our final version of the Engineering Documentation Report for the Green River Lock and Dam No. 3 project. This work was completed under Task Order No. 0008 (with modifications) of Contract No. W912QR-08-D-0005.

This report presents 30% designs and construction cost opinions for three remedial suites, as selected by USACE Louisville District (LRL). Each remedial suite focuses on repair of the rock-filled timber crib dam, lock chamber, mill race and rock shelf, and public safety.

Quality Assurance (QA) Review has been completed by LRL on this report, and the review comments have been addressed herein. Thank you for this opportunity to assist the Louisville District on this important project. Please contact us if you have any questions or require any additional information.

Sincerely,

STANTEC CONSULTING SERVICES INC.

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/rws

Enclosures: 1

cc: Jenni Reichard (CELRL-ED-M-A)
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Contract No. W912QR-08-D-0005
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Rochester, Kentucky

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Executive Summary

Green River Lock and Dam No. 3 (known locally as Rochester Dam) was built between 1833 and 1838, and consists of a fixed-crest overflow dam (rock-filled timber crib structure covered with derrick stone) and a stone masonry navigation lock (which has not been operated since 1981). The dam abuts a shallow rock outcrop (“rock shelf”) that also serves as part of the fixed-crest overflow control for the upper pool. A portion of the rock outcrop was excavated to create a “mill race” (the mill is no longer present).

Stantec was charged with the task of performing an assessment of the long term stability and integrity of Lock and Dam No. 3 and developing viable remedial options to a 30% design level with preliminary cost opinions. Major tasks included: historical document review, site geological assessment, geotechnical investigation, laboratory testing, underwater (i.e., diving) assessment and hydrographic survey, stability analyses and integrity assessment, development of preliminary remedial alternatives, and 30% design and cost opinions for three remedial options.

Initial project work included an extensive historical document review, subsurface exploration and laboratory soils testing (fall 2009) and underwater observations with hydrographic survey (summer 2010). With few exceptions, the findings were generally consistent with expectations for a facility of this type and age and the physiographic and geologic setting of this region. Chemical testing of select river sediment samples did not reveal concentrations of the targeted constituents above typical background levels for Kentucky soils.

A qualitative failure mode analysis was performed for the various components, to identify specific vulnerabilities to address during design of remedial options. The primary failure modes for the dam were related to collapse of the timber crib frame or loss of timber planking and subsequent unraveling of rock fill inside the timber frame. The primary failure mode for the lock was failure of the lock gates.

To address the most likely failure modes, Stantec developed preliminary remedial design alternatives for the rock-filled timber crib dam, masonry lock, and the rock shelf/mill race area with consideration given to maintaining crest elevation, ability to maintain the current pool, public safety, economics, and constructability. In February 2011, USACE Louisville District selected three remedial suites to advance to 30% designs and construction cost opinions. The three suites were generally packaged to provide lower, moderate, and higher cost options, consistent with the increasing project duration, complexity, and reliability of the remedial options.

Remedial Suite No. 1 addresses the rock-filled timber crib dam and the masonry lock but does not address the mill race and rock shelf area. In select areas, derrick stone on the face of the rock-filled timber crib dam will be replenished and then slush grouted in place. The upper lock gates will be buttressed with derrick stone. Remedial Suite No. 1 has an estimated project cost of approximately \$790,000.

Remedial Suite No. 2 is identical to Suite No. 1 for the dam, but with the addition of an upstream row of sheet piles and a reinforced concrete cap, constructed on the upstream sloping portion of the existing dam. To secure the lock chamber, a reinforced concrete bulkhead wall and splash pad will be constructed on the upper sill and keyed into the lock walls. Remedial Suite No. 2 has an estimated project cost of approximately \$3,300,000.

Remedial Suite No. 3 consists of a new cellular concrete dam, constructed upstream of the existing timber crib dam. The west end of the new dam will tie into the rock shelf. The new dam will extend across the upper lock approach and a new abutment will be constructed at the east bank. In the mill race area, a concrete overflow weir will be constructed along the same alignment as the new cellular dam. Remedial Suite No. 3 has an estimated project cost of approximately \$21,500,000.

Another alternative to repairing the facility is to take no action. The existing structures would remain in caretaker status or would be transferred to another owner through the disposition process without any modification or repair. Potentially vulnerable components of the structures have been identified, such as the lock gates and exposed areas of the timber crib frame of the dam. Without proper maintenance, the lock and dam cannot be expected to protect the pool adequately for an indefinite period of time. If no action were taken to remediate the structures, it is more likely that one or more of the identified failure modes could cause a loss of pool, although the timing of such an event cannot be predicted.

While all three of the selected remedial suites are viable alternatives to improve the facility, each has advantages and disadvantages, as well as uncertainties and risks. As future planning and design activities progress, the issues discussed herein, as well as the underlying assumptions should be reviewed and adjustments made if needed.

Engineering Documentation Report

Green River Lock and Dam No. 3

Contract No. W912QR-08-D-0005

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Rochester, Kentucky

1. Introduction

1.1. General

Lock and Dam No. 3 (Figure 1) on the Green River is located 108.5 river miles upstream from the Ohio River, in Rochester, Kentucky. Locally, the facility is also known as Rochester Dam. Built in the period 1833 to 1838 to enhance commercial navigation, the facility consists of a fixed-crest overflow dam (rock-filled timber crib structure covered with derrick stone) and a stone masonry navigation lock. The dam abuts a shallow rock outcrop ("rock shelf") that also serves as part of the fixed-crest overflow control for the upper pool. In 1848, a mill was constructed on the rock bluff at the left abutment, and a portion of the rock outcrop was excavated to create a "mill race" that channeled water to power the mill. In 1981, the U. S. Army Corps of Engineers (USACE) closed the lock to navigation and placed the facility in "caretaker" status. Since the 1960s, USACE has undertaken studies to evaluate the feasibility of a replacement lock and dam near the existing site, as well as studies to address the condition of the existing structures. However, no documented repairs have been performed on the lock since 1977 (replacement of the lower gates and installation of new operating mechanisms) and no documented repairs have been performed on the dam since 1966 (derrick stone placed on the timber face of the dam).

While water supply was not an authorized purpose for the facility, local communities and industry have become reliant on the pool retained by the lock and dam structures. Three water districts, serving over 46,000 people, have a total of five intakes in the pool. Also, a major industry in the area has a process water intake in the pool (USACE 2004; Gaines 2009). Beginning in 2001, an unofficial "Rochester Dam Coalition", led by local county Judges-Executive, has lobbied USACE and other federal officials to address concerns about the structural integrity of the lock and dam (Gaines 2009). Recent attention has been due in part to visual changes in the flow regime over the crest of the dam, which can be observed during periods of low flow. USACE compared photos from 2007 with photos from 1999-2000, and indicated that some stones appear to have been reworked or transported downstream (probably during high water events), thus altering the flow paths around the stones that can be observed during periods of low flow. However, based on visual observation by USACE Louisville District (LRL) during a 2007 site visit, the dam was not judged to be unstable (USACE 2007).

USACE-LRL performed a site visit and inspection on October 13, 2010 during a period of low flow. During this site visit, some areas of the dam (inaccessible during their 2007 visit) were visually inspected. A lack of derrick stone was observed in three areas near the crest on the downstream slope, thus exposing the timber crib frame in these areas. The derrick stone covering the contact between the timber crib dam and the rock shelf was observed to be in good condition. A memorandum from LRL, summarizing their site visit is included in Stantec (2010c), which is attached as Appendix H of this report.



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Rochester, Kentucky**



L&D 3 –
May1964_006.JPG
USACE-LRL

**Figure 1. Aerial Photograph of
Green River Lock and Dam
No. 3 in Rochester, Kentucky**

1.2. Scope of Work

Stantec is performing this work under Task Order No. 0008 of Contract No. W912QR-08-D-0005 for USACE-LRL. The task order was awarded on August 27, 2009. Modification No. 01 was issued September 21, 2009 to clarify schedule milestones from our original cost proposal. Modification No. 02 was issued on February 23, 2010 to provide for scope and schedule extensions related to delays (due to high river levels) in performance of diving and hydrographic survey work. The original task order and modifications are included in Appendix A.

The scope of work is to perform an assessment of the long term stability/integrity of Green River Lock and Dam No. 3 and to develop viable remedial options to a 30% design level with preliminary cost opinions. To date, Stantec has completed the following tasks for this project:

- Historical Document Review,
- Site Geological Assessment,
- Geotechnical Investigation including advancing 14 borings and collecting soil and sediment samples (October 2009),
- Laboratory Testing (physical and environmental testing of soil/sediment),
- Underwater (i.e., diving) Assessment and Hydrographic Survey (August 2010),
- Stability Analyses and Integrity Assessment, and
- Development of Remedial Options.

This report addresses the following tasks for this project:

- 30% Design and Cost Opinions for 3 Remedial Options

This report has been subjected to Independent Technical Review (ITR) by Stantec and Quality Assurance (QA) Review by LRL. Review comments and responses are included in Appendix F and have been addressed herein.

Three reports summarizing the data collected have been previously prepared and submitted by Stantec under Task Order No. 0008:

- Stantec Consulting Services Inc. (2010a). "Preliminary Findings Report." Prepared for the U.S. Army Corps of Engineers, Louisville District, March 11.
- Stantec Consulting Services Inc. (2010b). "Updated Preliminary Findings Report." Prepared for the U.S. Army Corps of Engineers, Louisville District, September 10.
- Stantec Consulting Services Inc. (2010c). "Stability and Failure Mode Analysis of Existing Conditions and Preliminary Remedial Design Alternatives." Prepared for the U.S. Army Corps of Engineers, Louisville District, December 10.

1.3. Engineering Documentation Report

Although this report is designated as the Engineering Documentation Report (EDR), LRL did not intend to have Stantec perform tasks to populate all sections of an EDR as outlined in ER 1110-2-1150, Appendix E (USACE 1999). However, it is our understanding that LRL does not intend to include this report as part of a more comprehensive EDR document. Stantec has attempted to conform the report to the outline and format in Appendix E where possible. It is beyond our scope to prepare sections such as E-5 (Status of Project Authorization), E-6 (Items of Local Cooperation and the Project Cooperation Agreement), E-12 (Economic Analysis), E-13 (Cost Allocation and Cost Sharing), and E-14 (Environmental Documentation and Coordination). Other sections outlined in Appendix E can be completed in part (e.g., E-4: Pertinent Data), but certain components are beyond the scope of work (e.g., benefit-to-cost ratio). If LRL eventually needed a comprehensive EDR (meeting all requirements of ER 1110-2-1150), additional efforts would be necessary.

Stantec's scope of work requires that this report include documentation of topics from previous deliverables, including long term stability, field work, and laboratory testing. In an effort to provide a comprehensive, stand alone document that represents the work performed in this task order, Stantec has included two previous reports (Stantec 2010b, 2010c) as Appendices G and H. Topics of interest in each of these reports are as follows:

- Stantec (2010b): site history, geologic setting, site exploration (including results of subsurface exploration, underwater observations, and hydrographic survey), laboratory testing, and proposed analysis methods.
- Stantec (2010c): lock wall stability, failure mode analysis, and preliminary remedial design alternatives.

2. Site Description

Green River Lock and Dam No. 3 was built by the Commonwealth of Kentucky between the years 1833 and 1838. After a 20-year period in which the state leased the works to the Green and Barren River Navigation Company (a private organization), the U.S. Government acquired possession of locks and dams on the Green and Barren Rivers on December 11, 1888 (Johnson 1974). Although the lock is no longer used for navigation, USACE still maintains the facility in "caretaker status". Key features of the facility are summarized in Table 1.

Table 1. Key features of Green River Lock and Dam No. 3

Feature	Statistic
Date started operation	1838
Location	37° 12' 49" North (NAD83) 86° 54' 01" West (NAD83)
Distance upstream of Ohio River	108.5 river miles
Length of pool upstream of dam	40.6 river miles
Length of dam spillway (estimated)	353 feet (260 feet timber crib and rockfill dam; 50 feet bedrock shelf; 43 feet excavated bedrock mill race)
Dam height (estimated)	27 feet (maximum)
Internal lock dimensions	35.8 feet by 137.5 feet
Lift (normal pool)	17 feet
Nearest Active Upstream Gage	NWS Gage RCHK2 (records headwater level 0.5 river miles upstream)
Nearest Active Downstream Gage	USGS Gage 03316500 at Paradise, Kentucky (records tailwater level 8.3 river miles downstream)
Maximum high water	414.2 feet (NGVD29) recorded 1937

2.1. Location

Green River Lock and Dam No. 3 is located at Green River Mile 108.5, immediately downstream of Rochester, Kentucky. The site is located about 0.2 miles downstream of the mouth of Mud River Figure 2. The lock property (east abutment) is in Ohio County, and can be accessed from Rochester Locks Lane, which branches off Kentucky Highway 369 just before reaching the Rochester Ferry landing on the Green River (approximately 0.5 miles upstream of the dam). The west abutment is in Muhlenberg County, and can be accessed from Kentucky Highway 70. The west abutment consists of a series of rock outcrops, and was the site of a mill from the mid-1800s to the early 1900s. Much of the river pool retained by the lock and dam is within Butler County or along the Butler County-Ohio County line.

2.2. Key Elevations and Dimensions

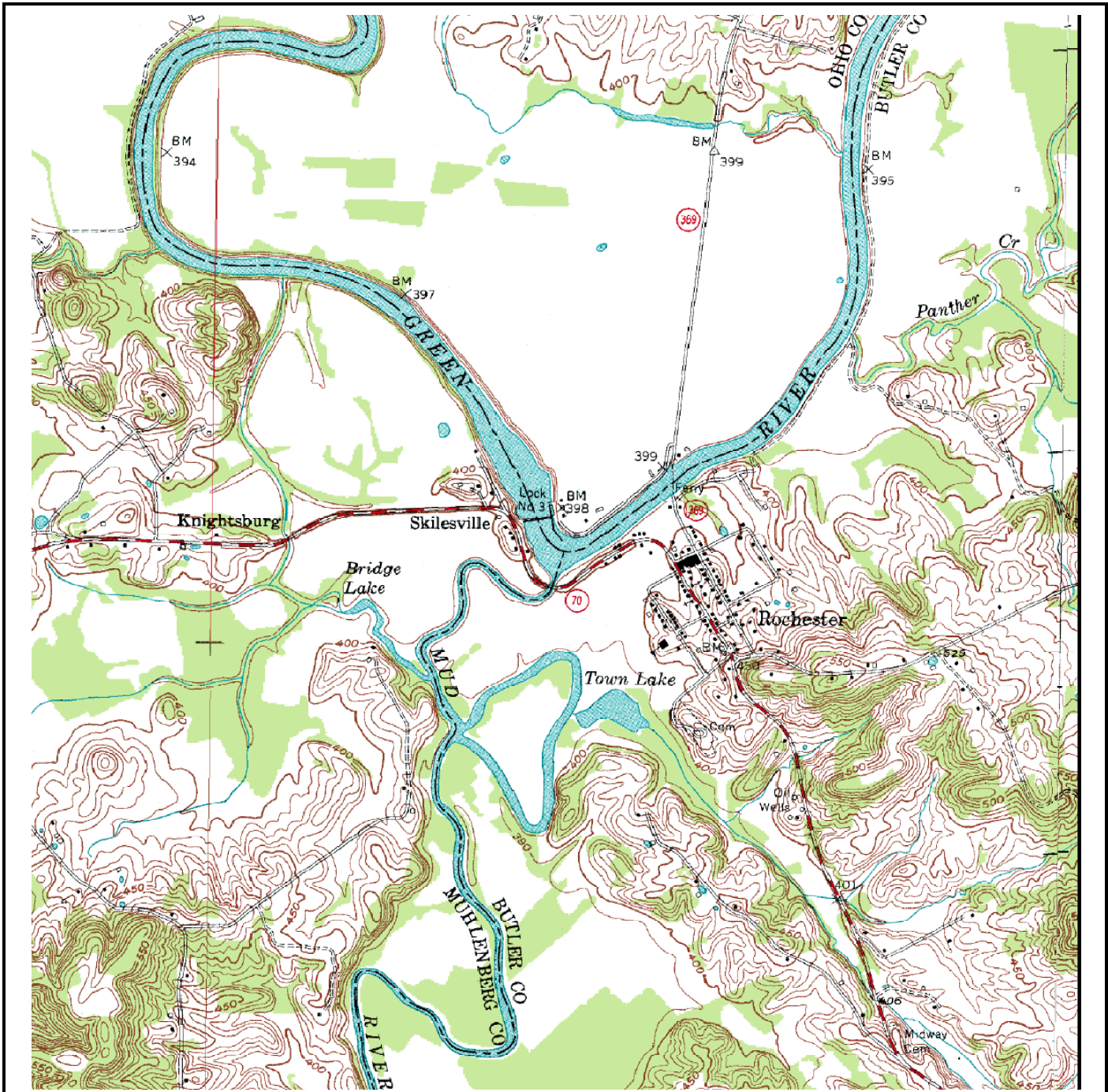
Historic drawings and other data sources are unclear on the vertical datum used when reporting elevations. Recent topographic survey of the site, performed by LRL, used the National Geodetic Vertical Datum of 1929 (NGVD29). The presence of derrick stone on the dam prevents an accurate survey of the existing crest of the dam, which could be compared to the crest elevation reported on historical drawings. Establishing the existing structure elevations (crest of dam, top of lock wall, etc.) are critical to moving forward with remedial designs. After bringing this issue to the attention of LRL, they provided the following direction:

- Based on LRL's 2009 topographic site survey, assign the top of lock wall an elevation of 390.40 feet (NGVD29). This typical value is based on multiple spot elevations measured along the top of the river wall.

- Assuming relative elevation differences shown on the Project Data Sheet (USACE 1995) are correct, calculate other key structure elevations based on the assigned top of lock wall elevation.
- The above logic was used to assign the key elevations for the facility, summarized below in Table 2.

Table 2. Key elevations at Green River Lock and Dam No. 3

Feature	Elevation (NGVD29)
Dam No. 3 Crest	380.7 feet
Upper Sill	373.4 feet
Lower Sill	358.1 feet
Top of Lock Wall (river wall and land wall)	390.40 feet
Upper Pool Gage Zero (i.e., datum)	373.36 feet
Lower Pool Gage Zero (i.e., datum)	358.11 feet
Dam No. 2 Crest (lower pool control)	363.7 feet



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Figure 2. Topographic Map of Green River
 Lock and Dam No. 3 Site



L&D 3 -
 May1964_006.JPG
 USACE-LRL

2.3. Structures

Green River Dam No. 3 (Rochester Dam) is an ungated, fixed-crest structure designed for continuous flow over a roughly 260-foot wide spillway. Based on the available historical documentation, the dam is founded on bedrock, and is about 27 feet tall (maximum) with a crest elevation of 380.7 feet (NGVD29). The dam functions as a gravity structure, wherein the weight of the dam resists sliding and overturning due to hydraulic pressures. Originally a rock-filled timber crib structure, the dam was repaired and rebuilt on numerous occasions, although documentation regarding design and repairs is limited.

The original dam, as described by Welch (1838) prior to its construction, was to be 260 feet long and 60 feet wide at its base. Each timber crib was to be 14 feet square, filled with rubble stone, and covered with timbers 6 to 8 inches thick. A similar design was planned for construction of the dams at Green River No. 4 and Barren River No. 1, while a narrower base (46 feet) and smaller cribs were planned for Green River No. 1. Unfortunately, there is no documentation to confirm that the dam was indeed built as described above. A report from the Secretary of War (1885) indicated that “The general construction of this dam is the same as at Nos. 1 and 2. The slope of the overfall is 16 degrees.”

In 1888, the U. S. Government acquired the locks and dams on the Green and Barren Rivers, which had deteriorated significantly since their original construction. An 1895 report from the Chief of Engineers, U. S. Army, reported that operations up to June 30, 1894, had included (among many things) rebuilding of Green River Dam No. 3. However, it is unclear how the reconstructed timber crib dam compared to the original structure.

The only cross section located for the dam is a schematic (Figure 3) from the USACE Project Data Sheet (1995). The basis for this schematic is unknown, and the elevation datum is unclear. The elevations shown in Figure 3 are inconsistent with the adjusted elevations shown in Table 2. The schematic indicates a timber crib structure bearing on rock, with a wedge of undefined fill material (possibly an earthen berm or cofferdam) along the upstream vertical face of the structure. If the scale is correct, the dam has a base width of approximately 81 feet (25 feet upstream of the crest, 56 feet downstream), a maximum height of approximately 27 feet, and a stepped downstream planking face that spills water onto a horizontal wooden apron that is slightly above the minimum lower pool elevation controlled by Green River Dam No. 2. The schematic does not reflect the derrick stone that was placed over the dam in 1966, nor does it show the accumulation of sediment upstream of the crest (which is apparent in recent photos taken when the river is extremely low).

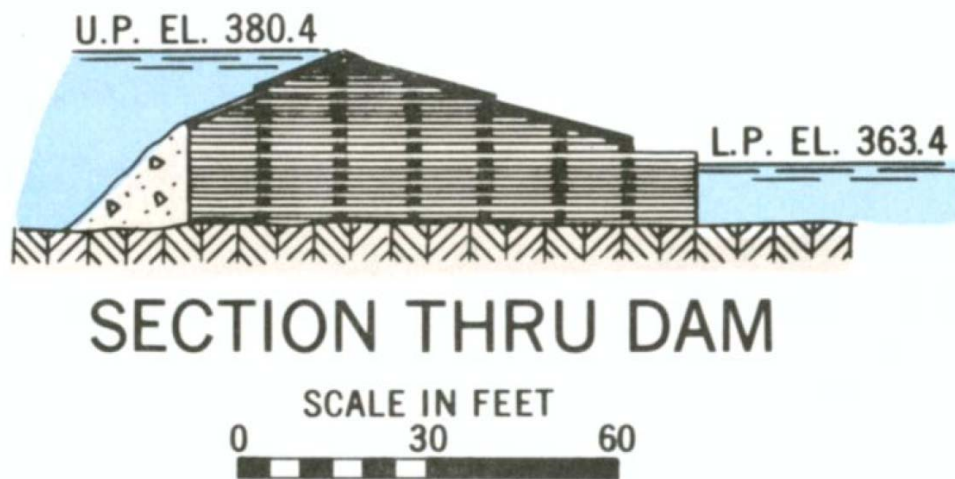


Figure 3. Schematic Cross Section of Dam No. 3 (USACE 1995)

On the east side, the timber crib dam abuts the river lock wall near the upper gates, although the nature of this connection is unknown and is now concealed by derrick stone. On the west side, the timber crib dam abuts a rock shelf that serves as part of the control section for the upper pool. The geometry of the contact between the dam and the shelf is unknown, and it is concealed by the river, derrick stone, and/or sediment. It is also unknown if/how the timber cribs were attached or notched into the rock shelf.

The approximate limits (in plan view) of the now concealed rock-filled timber crib dam are shown on Sheet C-001 in Appendix C. This geometry is based on limited evidence from historical drawings (including Figure 3 and a 1903 plan view that was based on a survey made in 1900) and photographs such as Figure 4. Both the 1903 plan view and Figure 4 would appear to confirm a slight bend (approximately 7 degrees) in the alignment of the dam at roughly 125 feet left of the river lock wall. Based on Figure 4, the left extent of the planking face appears to have extended over a portion of the rock shelf (a comparison with the 2009 LRL survey data indicates that such planking is no longer present at the site). Thus, this edge of the planking shown on Sheet C-001 may not be indicative of the location of the rock shelf-timber crib contact. While this information is useful in understanding the possible geometry of the structure, actual geometry is unknown and cannot be field verified, due to the presence of derrick stone, sediment, and/or debris.

The rock shelf surface is sandstone, and has a gently sloping downstream face similar in shape to the dam. Based on photos taken during low water (See Figure 5), a portion of the shelf (including the contact with the dam) appears to have a lower crest elevation than the dam, while another portion to the west is higher in elevation and thus is exposed during low water.

Between the above mentioned rock shelf and the rock bluff at the west (left) abutment is an excavated rock channel called the “mill race”. Recent photos taken during low water indicate that the mill race has a lower controlling/crest elevation than most of the dam and the rock shelf. Although not documented, it is presumed that this channel was excavated around 1848, when Brewer’s Mill was constructed at the west abutment. Photos taken in 1939 (e.g.,

Figure 4) would indicate that for some period of time, a wooden control section was constructed across the rock shelf and the mill race, presumably to provide a consistent crest elevation over the width of the river.

The navigation lock (Figure 6) abuts the dam on the right (east) side of the river. Construction began in 1833 and the lock was declared operable in 1838 (Oliver 1987; Crocker 1976). The original lock walls were built of sandstone masonry, laid in Louisville hydraulic cement (Johnson 1974). According to a letter from the Secretary of War (1885), the land lock wall was 9 feet thick on top at the gate recesses, 4 feet thick on top between recesses, and 12 feet thick at its base. The lock was founded on sandstone bedrock. The land wall included “counter-forks” at each end of the wall, 4 feet thick on top, extending into the bank 30 feet from the chamber face of the land wall. The river wall was 12 feet thick on top and both faces of the wall were vertical.

After sustaining significant damage during the Civil War, the river wall collapsed in 1887 (Johnson 1974). After the U. S. Government acquired possession of the locks on December 11, 1888, they initiated emergency reconstruction of the lock and dam. A cofferdam was constructed around the lock wall and the area was dewatered to aid in placing the new masonry. The lock reopened to navigation on November 10, 1890 (Crocker 1976).

Although historical documentation is very limited, it is assumed that the existing river wall is the product of the 1888-1890 reconstruction effort. Based on 2009 topographic survey data (above water only) provided by LRL, the existing river wall is approximately 12 to 13 feet thick on top at the gate recesses, and 8 feet thick on top between recesses. The land side (i.e., chamber side) face of the river wall is vertical, while the river side face is battered at approximately 0.15 horizontal to 1 vertical (0.15H:1V) at the recesses and 0.3H:1V between recesses.

The existing land wall is mostly concealed by the concrete paved esplanade, which was repoured in 1939 (based on historical photos) and could have been repaired or repoured again more recently. The esplanade measures roughly 35 feet by 200 feet in plan. The vertical chamber face of the land wall is sandstone masonry. Based on the available historical documentation, it would appear that the existing land wall is original (excepting for any undocumented repairs), with dimensions as described previously. Remnants of a metal hand railing are present along the length of the land wall.



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GreenNo3-1939_006_crop.jpg

USACE-LRL

**Figure 4. Photo Showing Planking Face
of Dam, Control Section, and
Rock Shelf (c. 1939)**



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Photos taken 8/1/2007 by LRL

Top: IMG4948.jpg

Bottom: IMG4689.jpg

Figure 5. Green River Lock and Dam No. 3, looking east from west abutment (top) and looking west from lock (bottom)

The lock chamber measures 35.8 feet by 137.5 feet in plan view, and has a vertical lift of 17.0 feet under normal pool conditions (USACE 1995). The chamber and lock approaches have accumulated a significant volume of sediment, vegetation, and debris since navigation ceased in 1981 (Figure 6). Although documentation is limited, it is likely that the miter gates have been repaired and/or replaced numerous times. According to Johnson (1984), new lower gates were installed in 1977 and electric gate operators were installed for both sets of gates, replacing the manually operated technology that had been used at the project since its original construction in the 1830s. The existing miter gates are heavily rusted and highly deteriorated, particularly the timber components. Much of the gates are not visible due to the accumulation of sediment, or because they are submerged in the river.

At the upstream end of the lock, a concrete upper guide wall (land side) and a concrete upper guard wall (river side) extend in the upstream direction. The upper guide wall is exposed for a length of roughly 100 feet, and due to heavy vegetation and sedimentation it is unclear if the wall terminates or is buried/concealed. Remnants of a metal hand railing are present along the length of the guide wall.

The concrete upper guard wall is separated from the river lock wall by a small (approximately 12-foot) gap. The upper guard wall extends roughly 175 feet upstream, is 4 feet wide on top, has a vertical inside face and a battered (approximately 0.3H:1V) outside (i.e., river side) face. The upstream end of the concrete upper guard wall adjoins an upper guard wall extension (roughly 240 feet long) that is constructed of vertical timber piling and horizontal timber rubbing beams. There are two lines of vertical piles, connected by short, battered lengths of piling. The upstream end of the extension is a rectangular “nose” made up of a steel Z-piling cell with a concrete top surface. Historical photos indicate that the extension was constructed in 1939. The river side of both the upper guard wall and upper guard wall extension has heavy vegetation and shallow water (due to the presence of soil/sediment near or above the normal pool elevation). It is possible that the alignment of these walls is along the original river bank prior to construction of the lock.

At the downstream end of the lock, a concrete lower guide wall extends along the bank roughly 100 feet. Based on historical drawings that are believed to be of this structure, the wall was constructed around 1912. The top width of the wall is 4 feet, the river face is vertical, and the land face is battered at approximately 0.25H:1V down to a base width of 10 feet. Below this, the concrete widens to 16 feet and rests on a concrete and rock filled timber cribbing foundation that may be a remnant from the previous wall. Remnants of a metal hand railing are present along the length of the concrete lower guide wall. The concrete lower guide wall adjoins a lower guide wall extension that is constructed of rock filled timber cribbing. Based on historical drawings that are believed to be of this structure, the wall was constructed around 1934, and is founded on older cribbing from the previous wall. The lower guide wall extension is highly deteriorated and several of the cribs lean significantly towards the river. The highest elevation of the cribbing is roughly 15 feet below that of the adjoining concrete wall. The cribbing remnants are visible over a length of roughly 120 feet downstream from the end of the concrete wall. The bank behind the cribbing wall is eroded significantly, exposing the back side of the cribs.



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Photos taken 8/3/2009

Top: HPIM0807.jpg

Bottom: HPIM0785.jpg

**Figure 6. Green River Lock and Dam
No. 3, Upper Gates (top) and
Lower Gates (bottom)**

When the lock was in operation, several USACE structures were present to the east of the lock chamber. Based on the Project Data Sheet (USACE 1995), remaining structures include a dwelling (former lockmaster house), garage, and office/warehouse. Topographic survey performed by USACE in 2009 also identified three structures in this area. Based on the 2004 Disposition Study (USACE 2004), the structures have intact wall and roof systems, but are otherwise in a state of disrepair and are continuing to deteriorate. The area surrounding these structures is heavily overgrown with vegetation and is not maintained.

A plan view illustrating the existing structures and topography at the site is included on Sheet C-001 in Appendix C.

3. Selection of Remedial Design Alternatives

3.1. Preliminary Remedial Design Alternatives

Preliminary design alternatives for Lock and Dam No. 3 were developed for the three major site components: (1) Rock-filled Timber Crib Dam, (2) Masonry Lock, and (3) Rock Shelf and Mill Race. The remedial options were developed with consideration given to maintaining crest elevation, stability of the lock and dam to maintain pool, public safety of the site with emphasis on the lock chamber, economics, and constructability.

With respect to potential loss of pool, the following failure modes were identified (Stantec 2010c) and were considered when developing remedial design alternatives: (1) Loss of Dam due to Downstream Scour, (2) Deterioration or Collapse of Dam Crest, (3) Failure of Lock Gates, (4) Erosion of Mill Race Area, (5) Sliding or Rotational Failure of Lock Wall, and (6) Failure of Dam due to Earthquake Loading.

Five preliminary remedial design alternatives were developed for each of the three major site components, as well as for public safety. Detailed descriptions along with preliminary drawings can be found in Stantec (2010c) in Appendix H.

3.2. Selection of Remedial Options

3.2.1. Review Meeting with LRL

On January 18, 2011, a meeting was held at the USACE Louisville District office with personnel from Stantec, LRL Structures Section, LRL Geotechnical and Dam Safety Section in attendance. The group was briefed on the project's objectives and current site condition prior to the discussion of the preliminary remedial design alternatives that were developed by Stantec (Section 3.1). The objective of the meeting was to arrive at three "remedial suites", covering a range of costs and complexity, based on how they address the following factors: maintaining crest elevation, stability of the lock and dam to maintain pool, public safety of the site with emphasis on the lock chamber, economics, and constructability.

During the general discussion of the remedial options, the group was encouraged to provide additional alternatives and/or combinations of alternatives that would be feasible. For example, one preliminary alternative for the dam was to install a line of upstream sheet piling and construct a concrete cap/slab over the entire surface of the dam (after removal of derrick stone). As a more economical alternative, the group suggested combining portions of two

lower cost alternatives: install a line of upstream sheet piling, construct a concrete cap on the upstream surface of the dam, and slush grout existing derrick stone on the downstream surface of the dam.

Options were evaluated for the dam, lock, mill race/rock shelf, and public safety. In an effort to implement a consistent evaluation and decision making process, the following factors were used to evaluate each option:

- *Impacts to Existing Structure:* Could the remedial measure cause harm to the existing structures that will remain in service? Is there increased potential for loss of pool during construction?
- *Constructability:* Is the method of construction proven and accepted in this setting? What methods are needed to perform adequate quality control and quality assurance? What is the anticipated construction duration, compared to the typically available construction season in this setting?
- *Cost:* What is the construction cost of the option?
- *Service Life and Reliability:* What is the anticipated service life of the remedial option? Can the controlling upper pool elevation (i.e., crest elevation) be reliably maintained over the service life?
- *Maintenance:* What are the expected long term maintenance requirements and costs?

At the conclusion of the meeting, general consensus had been reached on three preliminary remedial suites, each addressing the dam, lock, mill race/rock shelf, and public safety. The three suites were generally packaged to provide lower, moderate, and higher cost options. LRL senior management personnel were then asked to provide final review, modification, and approval of the three suites before moving forward with 30% design efforts.

3.2.2. Selected Remedial Suites

In a February 11, 2011, letter (Appendix B), LRL outlines the three remedial suites that were selected to advance to 30% designs and construction cost opinions. Public safety measures were discussed separately, as they can be applied in various ways to one or more of the three remedial suites. The following remedial suites were chosen by USACE-LRL:

1. Remedial Suite #1:

- a. Rock-filled Timber Crib Dam: Repair areas with displaced derrick stones and slush grout void spaces between and beneath stones.
- b. Masonry Lock: Buttress upper gates with stone.
- c. Mill Race and Rock Shelf: No remedial measure.

2. Remedial Suite #2:

- a. Rock-filled Timber Crib Dam: Install upstream sheet piling and upstream concrete cap. Also, on downstream face of dam, repair areas with displaced derrick stones and slush grout void spaces between and beneath stones.
- b. Masonry Lock: Construct concrete bulkhead wall at upper gates.
- c. Mill Race and Rock Shelf: No remedial measure.

3. Remedial Suite #3:

- a. Rock-filled Timber Crib Dam: Construct replacement cellular sheet pile dam immediately upstream of the existing dam.
- b. Masonry Lock: Extend replacement dam across the upper lock approach.
- c. Mill Race and Rock Shelf: Construct concrete weir across the mill race and rock shelf.

4. Public Safety:

- a. Install signage to deter public access to the site.
- b. Deter pedestrian access to the river lock wall.
- c. Create egress from the lock chamber, in the event that a pedestrian enters the chamber.

Upon receipt of the February 11, 2011 letter, Stantec collaborated with LRL to clarify specific aspects of each suite, in order to confirm a mutual understanding of the details needed to move forward with development of the design and cost opinions. These details are outlined below in Sections 4.1, 5.1, and 6.1.

4. Remedial Suite No. 1

4.1. Description

Remedial Suite No. 1 addresses the rock-filled timber crib dam and the masonry lock but does not address the mill race and rock shelf area. In select areas, derrick stone on the face of the rock-filled timber crib dam will be replenished, back to the approximate grade at which it was installed. In addition, the new derrick stone will be slush grouted in place. The upper lock gates will be buttressed with derrick stone and the lower gates will be pinned open to facilitate egress and to limit accumulation of sediment in the chamber. Drawings for Remedial Suite No. 1 are included on Sheets C-002 through C-007 in Appendix C, and additional details are presented below.

4.1.1. Rock-filled Timber Crib Dam

Dam No. 3 was designed as an overflow structure with a constant, fixed crest elevation; therefore, water is expected to flow uniformly over the dam. However, since original construction, the crest elevation has become irregular across the length of the dam, due to factors such as settlement, degradation of timbers, shifting of derrick stone, siltation upstream of the dam, etc. At lower pools, overflow has become concentrated in multiple small channels through the derrick stone, adjacent to the rock shelf (see Figure 7). Flow channels probably formed when derrick stone shifted during high flow events. During an October 2010 site visit, LRL personnel noted three locations on the dam face where derrick stone had been removed, exposing the timber cribbing. Approximate locations can be found on Sheet C-002 in Appendix C.

Remedial Suite No. 1 assumes placement of derrick stone in select areas, to replenish areas where stones have been removed during past flooding events. Such repair areas would be prepared by removing any debris or sediment, and then the new stones would be placed. Based on generally satisfactory past performance, the new stones would be sized similar to those currently in place (5- to 10-ton stones). Slush grout (or high slump concrete) would be placed beneath and around the stones to make multiple stones act more as a single mass (Sheet C-006). If the mass stays intact, this would effectively increase the size and weight of each unit, making it more resistant to movement during flood events. It is assumed that the slush grout (or concrete) mix design would include anti-washout admixture (AWA), so that the grout could be placed in flowing water (below some reasonable maximum velocity), thus limiting the need for water diversion. Some localized water diversion may be needed during slush grouting and will depend on repair location and flow conditions at the time of the repair.

Detailed surveying to identify areas where this treatment would be applied was beyond the scope of this study. For cost estimating purposes, the treatment was assumed for the three areas identified by LRL during their October 2010 site visit, as well as an additional 10 percent of the total surface area of the existing derrick stone (Sheet C-002).



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Top: GR LD3 8_30_1968 7.3.pdf
USACE-LRL

Bottom: HPIM1910.jpg - Stantec

Figure 7. Derrick stone dam face with limited overflow shown in 1968 (top), Current derrick stone placement with channelized flow in 2010 (bottom)

4.1.2. Lock Chamber

The lock chamber and lock approaches have accumulated a significant volume of sediment, vegetation, and debris since navigation ceased in 1981. In the absence of routine maintenance, the miter and quoin timbers have deteriorated, which can lead to improper load transfer to the lock walls and overstressing of gate components (pintle, pintle shoe, anchorages, etc.). Eventually, differential sediment and/or hydraulic loads, as well as deterioration of the gates themselves, could cause failure of the upper gates, resulting in the loss of Pool No. 3. Although the existing sediment on both sides of the upper gates acts as a buttress and a graded filter against seepage, it is subject to erosion during flood events and thus is an unreliable form of protection.

Remedial Suite No. 1 assumes installation of a rock buttress on both sides of the upper gates. No repairs to the upper gates are planned, although the condition of the upper gates is unknown. The first step of construction would consist of installing a temporary bulkhead in the upper lock approach to protect the pool during construction. Next, the sediment adjacent to the upper gates would be excavated down to the upper sill elevation (Sheet C-004). Excavation would need to be performed in alternating stages on the upstream and downstream sides of the gates, in order to prevent overloading in either direction. Removal of too much sediment on the upstream side could lead to the gates being forced open by the differential load in the upstream direction. After a sufficient footprint has been excavated, the upper gates would be buttressed on both sides with derrick stone (size equivalent to that used for the dam). The final grade of the stone was assumed to be 2H:1V in both upstream and downstream directions, with a top elevation of 385.4 feet of the upper gates (5 feet below the top of the lock wall). The five-foot drop off from the land lock wall to the top of the rock buttress is meant to be a deterrent to reaching the river lock wall on foot (although it is possible to walk along the narrow top of the gates).

The area around the lower gates would be excavated as needed to allow the gates to be recessed. The lower gates would then be pinned in the open position by securing them to the lock walls (Sheets C-003, C-005, and C-007). This will help to reduce sediment accumulation in the lock chamber, and also allow egress from the chamber in the event that a person accidentally enters the chamber. Finally, the existing railing along the land lock wall would be demolished and replaced with new railing (Sheet C-015) to deter pedestrians from accessing the lock. As additional deterrence to pedestrians and vehicles, new "No Trespassing" signage would be installed on the new railing, as well as along Rochester Locks Lane at the entry to the Federal Government property boundary.

4.1.3. Mill Race and Rock Shelf

Remedial Suite No. 1 does not include treatment of the mill race or rock shelf. LRL recognizes minor head cutting at the downstream end of the mill race; however, the controlling elevation of the mill race is a significant distance upstream of this head cutting. Therefore, the risk of pool loss through downward erosion or head cutting of the mill race or rock shelf is judged to be low, and remediation is not warranted for the less extensive, lower cost Remedial Suite No. 1. Similar to the east side of the river, "No Trespassing" signage would be installed at the parking lot/picnic area on the west side of the river, to deter pedestrians from accessing the mill race area.

4.2. Constructability and Performance

Construction of Remedial Suite No. 1 would likely be performed from the east bank and from barges in the lower pool. The derrick stone repair would be constructed using cranes and other support equipment on barges (Figure 8). Because the Green River is navigable to the downstream side of the dam, marine equipment could access the site from the Ohio River. Excavation and fill work related to the lock gates and rock buttress could be performed from the esplanade or from barge-mounted equipment in the lock chamber. An offsite, upland disposal area would be necessary for disposal of excavated sediment.

The likely performance of Remedial Suite No. 1 can be gauged based on the performance of the existing derrick stone. The derrick stone placed on the dam in 1966 has performed well over the past 45 years, although it is evident that some stones have been displaced. The proper size and weight of each stone is critical to withstanding potential extreme hydraulic forces during flood events. The relatively long, site-specific record of performance provides confidence that Remedial Suite No. 1 should perform in a similar fashion. However, as noted below, performance of the remediated dam is contingent on the condition and stability of the underlying timber crib dam, which is unknown.

This remedial suite has both advantages and disadvantages in regards to construction, economics, fulfilling design criteria, and durability (service life and maintenance needs).

Advantages of Remedial Suite No. 1 include:

- Work on the dam could be performed entirely from the downstream side. This may enable a marine contractor to access the site via the Ohio River, through Green River Lock Nos. 1 and 2, which are still open for navigation. This could lower mobilization costs and increase competition for the project.
- Based on past performance, the derrick stone on the dam has performed well and thus this option would be expected to perform in a similar fashion. The addition of the slush grout would enhance the stability of the derrick stone to some degree by interlocking multiple stones in a mass of grout.
- The slush grout may penetrate into the existing timber crib backfill, thus improving its resistance to seepage forces and/or erosion during flood events.
- The approach requires little removal or excavation of existing materials, and it is relatively easy to place new derrick stone and slush grout. Localized water diversion may be needed during slush grouting.
- The cost and duration to design and construct would be the lowest of the three remedial suites. The work could be performed during the dry summer and fall months of one construction season, reducing the risk to both the Contractor and the Owner in terms of exposure to potential flood events. The major construction tasks are simple and could be performed by a wide variety of marine contractors.
- Assuming the derrick stone remains in place, this option would require little to no maintenance.

- The rock buttress at the upper gates will be fairly resistant to erosion, and will provide minor support to the land lock wall (only adjacent to the upper gates).
- Pinning the lower gates in the open position will reduce the accumulation of sediment in the lock chamber. This will reduce loads on the river lock wall and will limit pedestrian access to the river lock wall. It also allows for egress from the lock chamber.



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scan0237.jpg - USACE-LRL

Figure 8. Example of derrick stone placement from barge in lower pool (Kentucky River Lock and Dam No. 11)

Disadvantages of Remedial Suite No. 1 include:

- The performance of the derrick stone is subject to the hydraulic loads imposed during potentially extreme flood events. In certain areas, it is evident that stones can be displaced during flood events.
- Any benefit from the slush grouting is contingent upon the grouted mass of stones acting as a unit. If the grouted mass were to crack, the system reverts to a network of individual derrick stones that perform much like the existing system. Given that the underlying timber and rockfill structure is basically untreated, there is a higher probability that future settlement in the rockfill could lead to cracking of the slush grout.
- This option continues to rely on the structural integrity of the existing rock-filled timber crib dam, but does not investigate or remediate the structure (other than surficial areas of derrick stone and slush grout placement). The structure may have defects and/or voids present that could continue to deteriorate and eventually collapse.
- Due to the high porosity of the derrick stone, it does not act to impound water. Therefore, the crest of the timber crib dam (assumed to be deteriorated and irregular) continues to control the upper pool elevation.
- Derrick stone is not commonly produced by quarries and would require a special order. The material costs could be relatively high, when compared to a more common, smaller rip rap/channel lining stone. However, given the flow regime over the dam and the performance history, smaller stone cannot be expected to stay in place.
- As for the lock chamber, the stone backfill could be displaced by hydraulic forces (e.g., during a flood event), thus removing support for the upper gates. Also, the derrick stone could settle because a portion of it would be founded on existing sediment (i.e., upstream and/or downstream of the upper sill). This may lead to future maintenance needs.
- Derrick stone is highly pervious; thus, leakage through or around the upper gates is not addressed by the rock buttress. No repairs are assumed for the upper gates (which have not been maintained since 1981), although exposing the buried surfaces of the gates could reveal the need for welding or other actions to eliminate obvious pathways for leakage. Although the accumulation of sediment along the upstream side of the gates would act as a natural graded filter, this is not a reliable means of seepage control. With regard to relying on the upper pool for water supply, leakage through the lock chamber could become a significant issue during periods of drought.
- While a small portion of the land lock wall would receive support from the derrick stone, the land lock wall would still be deficient in terms of stability design criteria. The derrick stone could impose additional loads on the river lock wall, although the area of loading is relatively small.

- Pinning the lower gates in the open position removes a level of redundancy in terms of pool security. The expected reduction of sediment in the lock chamber would reduce the stabilizing effect for the land lock wall. Leaving the upper gates in the closed position provides a potential avenue for pedestrian access to the river lock wall.

4.3. Construction Cost Opinion

For purposes of estimating contractor overhead, as well as escalation, a start date of May 2015 and a construction duration of 4 months was assumed. The assumed start date was provided by LRL, and the construction duration was assumed based on historical experience with similar projects in Kentucky. Per direction from LRL, a contracting mechanism similar to a Federal Multiple Award Task Order Contract (MATOC) was assumed, with a prime contractor acting as an administrator with limited duties, and a subcontractor who performs the construction work. Note that different assumptions regarding construction duration, start date, and/or contracting mechanism could significantly affect the cost opinion.

The construction cost opinion (i.e., project cost) for this remedial option is approximately \$790,000. The cost opinion derivation (from MCACES Second Generation, or MII) can be found in Appendix D. Electronic versions of the native MII files are included in Appendix D on the enclosed CD. In terms of cost, the most significant features of the work include the mobilization and demobilization of the floating plant (i.e., barges), derrick stone placement, and spoil disposal. Significant uncertainties for this option, with respect to cost (i.e., subject to change during future detailed design and/or during construction), are the quantities of derrick stone and slush grout necessary to treat the surface of the dam. Detailed surveying of the dam face was not performed for this investigation; therefore, an accurate estimate of the treatment area is not available. Transportation costs for hauling of spoil (i.e., dredged material) to an offsite, upland disposal site could be a significant percentage of the total cost. For cost estimating purposes, a haul distance was assumed, but an actual disposal site was not identified.

5. Remedial Suite No. 2

5.1. Description

Remedial Suite No. 2 addresses the rock-filled timber crib dam and the masonry lock but does not address the mill race area and rock shelf area. To create a uniform crest elevation, sheet piles will be driven to rock upstream of the existing rock-filled timber crib dam and a reinforced concrete cap will be constructed on the upstream sloping portion of the existing dam. The top elevation of the sheet piling and the concrete cap will be equal to the crest of the existing dam. The downstream face of the existing dam will receive treatment identical to Remedial Suite No. 1 (Section 4.1.1). That is, in select areas, derrick stone on the face of the rock-filled timber crib dam will be replenished, back to the approximate grade at which it was installed. In addition, the new derrick stone will be slush grouted in place.

A reinforced concrete bulkhead wall and splash pad will be constructed on the upper sill and keyed into the lock walls. The upper and lower lock gates will be pinned back into their recesses to deter pedestrians from accessing the lock chamber and to limit accumulation of sediment in the chamber. Drawings for Remedial Suite No. 2 are included on sheets C-008 through C-015 in Appendix C, and additional details are presented below.

5.1.1. Rock-filled Timber Crib Dam

As previously stated in Section 4.1.1, the dam was designed as an overflow structure with a constant, fixed crest elevation. Over time, the crest elevation has become irregular across the length of the dam, due to settlement, degradation of the timbers, shifting of the derrick stone, siltation upstream of the dam, etc. At lower pool elevations, the overflow has become concentrated in multiple small channels through the derrick stone, adjacent to the rock shelf.

Remedial Suite No. 2 restores the uniform crest elevation over the length of the timber crib structure by driving sheet piles immediately upstream of the existing structure and pouring a reinforced concrete cap between the sheet piling and the crest of the existing dam. The first step of construction would be to excavate derrick stone and rockfill along the corridor for the sheet piling, as these large materials would impede pile driving. Working in segments, a line of sheet piles would be driven through the remaining sediment (to bedrock) along the upstream face of the dam (Sheet C-008). An example of this type of construction, for a previous LRL project at Kentucky River Lock and Dam No. 5, is shown in Figure 9.

The area between the piles and the existing timber crib crest would then be excavated to expose the original timber crib structure. This surface would be inspected and prepared for the overlying concrete cap placement after the area is dewatered. This preparation may include repair of defects and/or backfilling of voids with stone or concrete, although any concealed defects may remain untreated. A concrete cap, with a horizontal crest at elevation 380.7 feet, would be placed between the piling and the existing crest on the timber crib structure (Sheet C-014). In the event that the entire cap area cannot be dewatered, the concrete would be placed in two pours. The lower pour would be placed underwater via the tremie method, and would help to reduce inflow by sealing the base. Any remaining seepage could be controlled by pumping, to allow the upper pour to be placed in-the-dry using conventional methods. Reinforcing steel would be installed in the upper portion of the cap, not only to reinforce the new concrete, but also in an effort to connect the new concrete to the timber crib structure and to the new sheet piling. After the concrete cures, the sheet piles would then be cutoff to the crest elevation.

Based on historical LRL drawings for similar repairs on the Kentucky River, special treatments at both ends of the sheet piling wall are incorporated. In an apparent effort to reduce seepage potential where the sheet piling must abut steeply inclined rock surfaces and/or the lock wall, LRL has typically added a wider zone that receives full depth concrete (i.e., to rock). The fluid concrete can then conform to the sloping surfaces much better than the sheet piling, which would tend to leave stair-stepped seepage “windows” where one side of each pile refuses on rock or a structure. The sheet piling in these areas would be driven, temporarily braced, and then excavated on the interior to clean rock prior to concreting (in the wet, via the tremie method). Refer to Figure 9 for an example of this construction element, and Sheets C-010, C-011, and C-014 for the implementation of this approach for Remedial Suite No. 2.

PZ22 sheet piling was assumed to be driven to rock for this remedial suite. Design calculations (Appendix E), based on guidance in the “Steel Sheet Piling Design Manual” (United States Steel 1984), indicate that PZ22 sheets would have sufficient capacity during excavation and dewatering, as well as during concreting.

Due to limited historical and geotechnical data immediately upstream of the dam, the rock line along the upstream side of the existing timber crib dam is uncertain; therefore, the estimated sheet piling quantities are uncertain. Further, portions of the bedrock at the site were found to be soft and/or weathered (Appendix G); thus, the refusal depth for the piling is more uncertain than if the rock were consistently much harder than the overlying soils.

Regarding possible repair or treatment of the upstream face of the timber crib dam, this area is currently inaccessible and its condition is unknown. Exploration of this area prior to removal of the overlying derrick stone would be quite difficult. For cost estimating purposes, 25 percent of the timber crib surface area (beneath the footprint of the reinforced concrete cap) was assumed to require backfilling with concrete to a depth of 2 feet.

As in Remedial Suite No. 1 (Section 4.1.1), derrick stone would be placed in select areas to restore the approximate original grade on the downstream face of the existing dam. As a cost savings measure, the derrick stone excavated to facilitate upstream sheet pile installation could be reused to repair the downstream face of the dam. The new derrick stone would also be slush grouted into place. Detailed surveying to identify areas where additional derrick stone would be required was beyond the scope of this study. For cost estimating purposes, the derrick stone/slush grout treatment was assumed for the three areas identified by LRL during their October 2010 site visit, as well as an additional 10 percent of the surface area of the existing stone as highlighted on Sheet C-008.



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Top: scan0006.jpg - LRL

Bottom: scan0055.jpg - LRL

Figure 9. Example of Upstream Sheet Piling Construction (Kentucky River Lock and Dam No. 5)

5.1.2. Lock Chamber

As discussed in Section 4.1.2, the chamber and lock approaches have accumulated a significant volume of sediment, vegetation, and debris since navigation ceased in 1981. In the absence of routine maintenance, the miter and quoin timbers have deteriorated and are possibly resulting in overstressing and improper load transfer to the lock walls. It would be possible for the lock gates to fail, under critical load conditions, resulting in the loss of Pool No. 3.

Remedial Suite No. 2 assumes installation of a concrete bulkhead wall on the upper sill inside of the existing upper gates. The first step in construction would consist of installing a temporary bulkhead in the upper lock approach to protect the pool during construction. Next, the sediment adjacent to the upper gates would be excavated down to the upper sill elevation (Sheet C-012). Excavation would need to be performed in alternating stages on the upstream and downstream sides of the gates, in order to prevent overloading in either direction. Removal of too much sediment on the upstream side could lead to the gates being forced open by the differential load in the upstream direction.

Next, a reinforced concrete bulkhead wall (Figure 10) would be constructed on the upper sill. The wall would be keyed into the lock walls and the upper sill as shown on Sheet C-015. The top elevation of the bulkhead wall (385.4 feet) is located between the crest elevation of the dam (380.7 feet) and the lock wall elevation (390.4 feet). The lower elevation of the bulkhead wall (compared to the lock gates) would help to limit the height of sediment accumulating in the lock chamber, thus providing a deterrent to individuals attempting to access the river lock wall by walking across the sediment in the chamber. The higher elevation of the bulkhead wall (compared to the dam) would limit the frequency of flow through the lock chamber, thus limiting the potential for scour along the inside toe of the land and river lock walls. A reinforced concrete splash pad would be constructed on the portion of the upper sill downstream of the new bulkhead wall (Figure 10). The splash pad provides erosion resistance against overtopping flows.

After construction of the bulkhead wall, the upper gates would then be pinned in the open position by securing them to the lock walls (Sheets C-012 and C-015) using four steel tiebacks (Figure 11) which are comprised of a W-steel section, a steel plate, and an anchor rod (Sheet C-015).

The area around the lower gates would also be excavated as needed to allow the gates to be recessed. The lower gates would then be pinned in the open position by securing them to the lock walls (Sheets C-012 and C-015). This will help to reduce sediment accumulation in the lock chamber, and also allow egress from the chamber in the event that a person accidentally enters the chamber. Finally, the existing railing along the land lock wall would be demolished and replaced with new railing (Sheet C-015) to deter pedestrians from accessing the lock. As additional deterrence to pedestrians and vehicles, new "No Trespassing" signage would be installed on the new railing, as well as along Rochester Locks Lane at the entry to the Federal Government property boundary.



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Top: P0004975.jpg – Stantec

Bottom: scan0263.jpg – LRL

Figure 10. Examples of Concrete Bulkhead
 Wall Construction (Kentucky River
 Lock and Dam Nos. 8 (top) and 11
 (bottom))



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Top: P0002312.jpg – Stantec

Bottom: P0002320.jpg – Stantec

Figure 11. Example of Steel Tiebacks to Secure Gates (Kentucky River Lock and Dam Nos. 8 and 9)

5.1.3. Mill Race and Rock Shelf

Remedial Suite No. 2 does not include treatment of the mill race or rock shelf. LRL recognizes minor head cutting at the downstream end of the mill race; however, the controlling elevation of the mill race is a significant distance upstream of this head cutting. Therefore, the risk of pool loss through downward erosion or head cutting of the mill race or rock shelf is judged to be low, and remediation is not warranted for the moderately extensive, moderate cost Remedial Suite No. 2. Similar to the east side of the river, “No Trespassing” signage would be installed at the parking lot/picnic area on the west side of the river, to deter pedestrians from accessing the mill race area.

5.2. Constructability and Performance

Construction of Remedial Suite No. 2 would likely be performed from the east bank and from barges in the upper pool. The upstream sheet piling and concrete cap would be constructed using cranes and other support equipment on barges. Because the lock is inoperable, marine equipment would have to be mobilized by truck and deployed to the river at a dock or boat ramp. The relatively small quantity of concrete needed could be supplied by typical ready-mix plants (depending on travel time). Excavation work related to the lock gates and bulkhead wall construction could be performed from the esplanade or from barge-mounted equipment in the lock chamber. An offsite, upland disposal area would be necessary for disposal of excavated sediment.

Installing upstream sheet piling and a partial or full concrete cap over the dam has been performed by LRL several times over the past 55 years on Kentucky River dams, which are similar to those on the Green River. As observed in recent (2007) above and below water assessments (Stantec 2008), sheet piling placed in the 1950s through the early 1970s now have seepage pathways as a result of corrosion and/or loss of filler material at irregular interfaces with rock or the lock walls. Corrosion is typically focused near the tops of the piling, where oxidation is more common due to fluctuating water levels during times of severe drought. The upstream concrete caps have settled relative to the sheet piling, and horizontal gaps have opened between the top of the piling and the top of the concrete. Improved designs, implemented during the late 1970s through the 1990s, have fared better, primarily due to better subgrade preparation beneath the concrete cap and more robust connection between the sheet piling and the concrete reinforcement (which limits differential movement). Widespread corrosion was not observed for piling installed as long ago as 1976.

This remedial suite has both advantages and disadvantages in regards to construction, economics, fulfilling design criteria, and durability (service life and maintenance needs).

Advantages of Remedial Suite No. 2 include:

- Construction and performance of this approach has proven to be successful in similar settings. As discussed above, similar dams on the Kentucky River have been repaired by LRL using this approach.
- The sheet piling and concrete cap provides a consistent, durable crest, and will reduce the potential for areas of concentrated or channelized flow over the dam.

- The sheet piling will reduce the potential for channelized flow through the timber cribbing, thus reducing the potential for accelerated deterioration and collapse of the timber crib frame.
- Based on past performance, the derrick stone on the dam has performed well and thus the derrick stone in this option would be expected to perform in a similar fashion. The addition of the slush grout would enhance the stability of the derrick stone to some degree by interlocking multiple stones in a mass of grout.
- The concrete bulkhead wall does not rely on the upper gates for stability or seepage control.
- The bulkhead wall will not be susceptible to erosion during flood events, when compared to stone fill. In general, the bulkhead wall would require little maintenance during its service life.
- Pinning the gates in the open position will reduce the accumulation of sediment in the lock chamber. This will reduce loads on the river lock wall and will limit pedestrian access to the river lock wall. It also allows for egress from the lock chamber.
- Derrick stone removed from the upstream face of the dam could be used to replenish areas on the downstream dam face. This provides a cost savings in terms of both repair materials and a reduction in disposal costs for excavated materials.
- The work could be performed during the summer and fall months of one construction season, reducing the risk to both the Contractor and the Owner in terms of exposure to potential flood events. The major construction tasks are simple and could be performed by a wide variety of marine contractors.

Disadvantages of Remedial Suite No. 2 include:

- The approach requires excavation and disposal of materials at an offsite location. Depending on the excavation quantities and the haul distance, this could be a significant project cost. Removal of the large derrick stone may require specialized crane attachments, and production rates might be rather low.
- Water diversion and dewatering will be necessary to construct the concrete cap and the bulkhead wall. This approach does have the inherent benefit of using the upstream sheet piling for water diversion, but the left and right ends of each section of work would also require temporary diversion.
- Differential movement between the sheet piling (bearing on rock) and the concrete infill (bearing on the timber crib dam) could lead to horizontal and/or vertical gaps at the interface. Pinning the tops of the piles to the concrete mass will help, but might not completely prevent such movement.
- Seepage windows are likely at each end of the sheet piling wall, due to an irregular pile tip profile at the river lock wall and the steeply sloping rock shelf

interface. As described above, special tie-in features are necessary to control seepage pathways at these locations.

- The difficulty of exploring and characterizing the condition of the timber crib structure during the design phase creates uncertainty, and may lead to more conservative designs and/or greater potential for concealed conditions and resulting change orders during construction.
- Stability and integrity of the upstream concrete cap is still dependent on support from the underlying timber crib structure. However, with the ability to inspect and treat this portion of the structure, the reliability of this support is improved.
- Derrick stone is not commonly produced by quarries and would require a special order. The material costs could be relatively high, when compared to a more common, smaller rip rap/channel lining stone. However, given the flow regime over the dam and the performance history, smaller stone cannot be expected to stay in place.
- While a small portion of the land lock wall would receive support from the concrete bulkhead wall, the land lock wall would still be deficient in terms of stability design criteria.
- Pinning the lower gates in the open position removes a level of redundancy in terms of pool security. The expected reduction of sediment in the lock chamber would reduce the stabilizing effect for the land lock wall. More frequent flows through the lock chamber could lead to scouring at the toe of the land and river lock walls. However, this risk is mitigated by the fact that the tailwater would be elevated before water overtops the bulkhead wall.
- As with Option 1, the derrick stone has proven its performance over the years, but the performance of the derrick stone is subject to hydraulic loads imposed during potentially extreme flood events.
- Any benefit from the slush grouting is contingent upon the grouted mass of stones acting as a unit. If the grouted mass were to crack, the system reverts to a network of individual derrick stones that perform much like the existing system. Given that the underlying timber and rockfill structure is basically untreated, there is a higher probability that future settlement in the rockfill could lead to cracking of the slush grout.

5.3. Construction Cost Opinion

For purposes of estimating contractor overhead, as well as escalation, a start date of May 2015 and a construction duration of 8 months (one construction season) was assumed. The assumed start date was provided by LRL, and the construction duration was assumed based on historical experience with similar projects in Kentucky. Per direction from LRL, a contracting mechanism similar to a Federal Multiple Award Task Order Contract (MATOC) was assumed, with a prime contractor acting as an administrator with limited duties, and a subcontractor who performs the construction work. Note that different assumptions regarding

construction duration, start date, and/or contracting mechanism could significantly affect the cost opinion.

The construction cost opinion (i.e., project cost) for this remedial option is approximately \$3,300,000. The cost opinion derivation (from MCACES Second Generation, or MII) can be found in Appendix D. Electronic versions of the native MII files are included in Appendix D on the enclosed CD. In terms of cost, the most significant features of the work include concrete and sheet piling for the dam repair. Significant uncertainties with this option, with respect to cost (i.e., subject to change during future detailed design and/or during construction), are the quantity of sheet piling, the potential need for surficial repairs (assumed to entail backfilling with concrete) to the timber crib dam, and potential obstructions to sheet pile driving

Due to limited historical and geotechnical data immediately upstream of the dam, the rockline along the upstream side of the existing timber crib dam is uncertain. The rockline defined by the borings is representative of an alignment roughly 50 feet upstream of the proposed sheet piling alignment for Remedial Suite No. 2. Therefore, there is some uncertainty associated with extrapolation of the boring data to the sheet piling alignment, although it is reasonable to assume that any difference should be relatively modest, with possible exception of the steeply sloping area adjacent to the rock shelf. This uncertainty affects the estimated sheet piling quantities.

The rockline is not well defined near the rock shelf, where it slopes steeply. Further, portions of the bedrock at the site were found to be soft and/or weathered (Appendix G); thus, the refusal depth for the piling is more uncertain than if the rock were consistently much harder than the overlying soils. Regarding possible repair or treatment of the upstream face of the timber crib dam, this area is currently inaccessible and its condition is unknown. Exploration of this area prior to removal of the overlying derrick stone would be quite difficult. Therefore, an accurate estimate of the treatment area is not available.

Uncertainty also exists regarding potential derrick stone obstructions to sheet pile driving. It is assumed that a substantial thickness of sediment had accumulated along the upstream face of the dam prior to derrick stone placement in 1966 (the dam would have been roughly 130 years old at that time). Pre-excavation down to elevation 370 feet (approximately 11 feet below historical crest elevation, see Sheet C-014) is designed to remove the large majority of derrick stone that may be present along the sheet piling alignment. However, it is possible that derrick stone could be present below this pre-excavation level, and could hinder sheet pile driving efforts. Further, it is possible that fill or other materials placed immediately upstream of the dam during original construction could contain large particles that could obstruct sheet pile driving.

6. Remedial Suite No. 3

6.1. Description

Remedial Suite No. 3 addresses the rock-filled timber crib dam, the masonry lock, and the mill race area. To create a uniform crest elevation, a new cellular concrete dam will be constructed upstream of the existing timber crib dam. The west end of the new dam will tie into the rock shelf. The new dam will extend across the upper lock approach, which eliminates the need for remediation of the upper gates or the lock chamber. A new abutment will be constructed at the east bank, upstream of the existing esplanade.

In the mill race area, a concrete overflow weir will be constructed along the same alignment as the new cellular dam. The crest of the weir would be equal to the controlling elevation of the entry to the mill race, which is assumed to be a few feet lower than the crest elevation of the dam. Drawings for Remedial Suite No. 3 are included on sheets C-016 through C-022 in Appendix C, and additional details are presented below.

6.1.1. Rock-filled Timber Crib Dam

Instead of attempting to repair and maintain the 172-year-old dam, it may be more cost effective and reliable to leave the existing dam in place and build a new dam immediately upstream of the existing structure, tying into the rock shelf on the west side and extending across the upper lock approach and constructing a new abutment at the east river bank. While many different styles of dam are possible, a concrete-filled cellular sheet pile structure is assumed for this site. This style of replacement dam has been constructed by LRL at Green River Lock and Dam Nos. 1 and 2 and by the Commonwealth of Kentucky at Kentucky River Lock and Dam Nos. 9 (completed 2010) and 3 (in progress). Examples of this style of dam are shown in Figure 12.

Suite No. 3 includes a new concrete-filled cellular sheet pile structure, comprised of alternating main cells and arc cells, that will replace the existing timber crib dam. The crest of the replacement dam would be constructed at the design crest of the timber crib dam (380.7 feet) thereby limiting changes to the hydraulic signature of the river.

Extending from the rock shelf to the east bank, a concrete-filled cellular dam will be constructed along an alignment approximately 60 feet upstream of the existing structure (Sheet C-016). Construction of each circular main cell in the river would begin by pre-excavating much of the sediment in the footprint. A circular, steel template would be placed in the river to guide placement of the flat sheet piles and to support the cell as it is constructed (Figure 12). After the cell is closed, the interior would be excavated to a clean rock surface. The interior would then be filled with unreinforced mass concrete using the tremie (i.e., underwater placement) method. The template would be withdrawn as the tremie concrete is placed. The upper portion of the cell would be dewatered and reinforcing steel would be installed. A concrete cap would be placed in-the-dry using conventional methods. After two adjacent main cells are completed, the interconnecting arc cell would be constructed in a similar manner. Construction of Cell 5 would also require demolition (to full depth) of a portion of the existing upper guard wall (Sheet C-019). Derrick stone (5- to 10-ton stones) will be placed at the downstream toe of the new dam to protect against scour.

The cellular dam layout was selected from standard layouts using 30-degree Y pile connectors to join the main cells to the arc cells. The cells were sized to meet USACE design criteria for gravity dams, in accordance with EM 1110-2-2100 (USACE 2005). Refer to Appendix E for stability calculations for the required load scenarios. The critical cross section was in the area of the upper lock approach, where the depth to rock is greatest along the alignment of the cells. In order to reduce uncertainty in the rockline along the dam alignment, future phases of design may consider additional borings. The shear strength along the base of the cells is a critical parameter when evaluating sliding stability. Per the scope of work, the shear strength used herein was estimated based on typical, published values for the general rock types observed in the geotechnical exploration. Future phases of design must include derivation of site-specific shear strength parameters. Laboratory direct shear testing of large

diameter rock core samples is recommended, along with geologic field study of rock outcrops to characterize large scale rock mass and bedding plane features.

After Cell 1 is completed, construction could begin on the West Closure Cell (WCC), which ties into the rock shelf. The WCC is a rectangular structure comprised of upstream and downstream lines of PZ sheet piling driven to rock. The cell would require temporary bracing for support prior to and during concrete placement. Similar to the main cells and arc cells, the WCC would receive unreinforced concrete infill (placed via tremie method) and a reinforced concrete cap (Sheet C-018). The crest elevation of the WCC would vary, transitioning to match the surface of the rock shelf. Derrick stone would be placed along the downstream toe of the WCC. Due to the steeply inclined rock surface in this area, future design efforts should consider additional soundings, borings, and/or hydrographic surveys to better define the top of sediment and the rockline.



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Top: picture 012.jpg – Stantec

Bottom: Stantec

Figure 12. Example of Cellular Sheet Pile Dam Construction (Kentucky River Lock and Dam Nos. 9 (top) and 3 (bottom))

The new east abutment would consist of an East Abutment Cell (connected to Cell 5), a Sheet Pile Cutoff Wall (extending into the bank), and a Training Wall Cell (between the East Abutment Cell and the upstream end of the Land Lock Wall). The purpose of the Sheet Pile Cutoff Wall is to lengthen the seepage pathway around the abutment of the new dam, and to guard against a flanking failure. The Training Wall Cell provides full depth retention of the river bank in an area that is currently retained by the upper guide wall, which is assumed to be inadequate when this area is subjected to downstream water levels and scour conditions. The Training Wall Cell has been sized assuming it acts as a gravity retaining structure, similar to the cells of the new dam. Refer to Appendix E for design calculations for this component. Note that in future phases of design, it is likely that this component could be refined or redesigned to be more efficient and thus more cost effective.

Portions of the upper guide wall would be demolished to full depth to allow construction of the new abutment. Construction of both cells assumes the need for temporary bracing to resist external soil loads until concrete infill can be placed. Each cell would be filled with unreinforced concrete infill below water, and reinforced concrete above water. The top elevation of both cells will be approximately equal to the top of lock wall elevation of 390.4 feet.

A railing would be constructed along the riverside edge of the East Abutment and Training Wall Cells as a public safety measure. As additional deterrence to pedestrians and vehicles, new “No Trespassing” signage would be installed on the new railing, as well as along Rochester Locks Lane at the entry to the Federal Government property boundary.

6.1.2. Lock Chamber

Remedial Suite No. 3 assumes that the replacement dam will extend across the upper lock approach thus eliminating the need for repair of the lock chamber. The land lock wall still serves to retain a portion of the river bank, but it is no longer integral to protecting the upper pool. Once the replacement dam is complete, the upper and lower sills will be dredged as needed to allow for the upper and lower gates to be pinned in the open position (Sheets C-017 and C-022). As in the other remedial suites, the existing railing along the land lock wall would be demolished and replaced with new railing (Sheet C-022) to deter pedestrians from accessing the lock. As additional deterrence to pedestrians and vehicles, new “No Trespassing” signage would be installed on the new railing.

6.1.3. Mill Race Area

Between the above mentioned rock shelf and the rock bluff at the west (left) abutment is an excavated rock channel called the “mill race” (Figure 13). Visual observations during low water confirm that the mill race has a lower controlling/crest elevation than most of the dam and the rock shelf. Although not documented, it is presumed that this channel was excavated around 1848, when Brewer’s Mill was constructed at the west abutment. Photos taken in 1939 would indicate that for some period of time, a wooden control section was constructed across the dam, rock shelf and the mill race, presumably to provide a consistent crest elevation over the width of the river. Note that the geometry of the mill race channel could not be surveyed during this study due to swift currents and related safety concerns. Future phases of design should include efforts to better define this area, in order to reduce uncertainty during construction.



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**Figure 13. Mill Race at Green River Lock
and Dam No. 3**



Top: LRL

Bottom: HPIM1959.jpg – Stantec

Remedial Suite No. 3 assumes the addition of a concrete overflow weir structure within the mill race (Sheet C-020). The general size and shape of the weir was designed to meet USACE design criteria for gravity structures, following similar logic and assumptions as used for the cellular dam. Refer to Appendix E for stability calculations. However, the hydraulic performance of the structure has not been evaluated or compared to USACE design criteria. Such efforts should be included in future phases of design. The crest elevation of the weir would be set equal to the current controlling mill race elevation, such that the hydraulic signature of the mill race is preserved.

In order to construct the overflow structure, water will have to be diverted from the mill race area. This would be accomplished by constructing a stone or earthen berm (cofferdam) around the work area. The work area would be maintained in a dewatered condition by pumping out any seepage water that enters the area.

The loose rock and debris present in the weir footprint would be removed to provide a clean, sound foundation for the concrete structure. Temporary formwork for the weir would be constructed, reinforcing steel would be placed, and conventional cast-in-place concrete would be poured.

The lower crest elevation of the weir (compared to the crest of the dam) will ensure that during typical conditions, water will flow through the mill race. This will act as a deterrent to pedestrians attempting to walk across the mill race from the left abutment rock highwall to access the new dam. Additionally, "No Trespassing" signage would be installed at the parking lot/picnic area on the west side of the river, to deter pedestrians from accessing the mill race area.

6.2. Constructability and Performance

Construction of Remedial Suite No. 3 would likely be performed from the east bank and from barges in the upper pool. The cellular dam would be constructed using cranes and other support equipment on barges. Because the lock is inoperable, marine equipment would have to be mobilized by truck and deployed to the river at a dock or boat ramp. Due to the large quantities of concrete necessary for the new dam, it is possible that a temporary batch plant could be setup at or near the site, although supply from typical ready-mix plants may also be a viable option (depending on travel time). Excavation work related to the lock gates could be performed from the esplanade or from barge-mounted equipment in the river. Due to the large excavation quantities associated with construction of the new dam, a significant offsite, upland disposal area would be necessary. The mill race area could be accessed from the west side of the river, but it is more likely that access would be from the river, as much of the contractor's operation would be staged from the east bank and progress to the west as the cellular dam is completed.

As mentioned previously, cellular sheet pile dams are commonly used for low head, navigation dams. With respect to cellular sheet pile dams with granular infill, EM 1110-2-2607 states the following: "The life of these cellular structures will likely be controlled by the longevity of the sheet piling, which can exceed 50 years under favorable conditions." Because Remedial Suite No. 3 consists of cellular structures with concrete infill, the product service life is not dependent on the life of the sheet piling. Further, this alternative is not dependent on any existing structure, such as the timber crib dam. Therefore, the product service life of the concrete dam is anticipated to be in excess of 50 years. A concrete-filled cellular dam on the Muskingum River (Ohio) was constructed in 1952 and is still performing

well. A similar dam on the same river, constructed in 1946 with granular infill and a concrete cap, is still in service but shows more signs of deterioration due to loss of granular infill via sheet pile corrosion.

Advantages of Remedial Suite No. 3 include:

- Constructing the new dam independent of the existing structure provides greater certainty of execution, a more durable/robust structure, and less maintenance.
- Only minor flow diversion required for construction of the replacement dam, and can be provided using the cellular sheet pile structure itself.
- The concrete cells provide a consistent, durable crest, and will reduce the potential for areas of concentrated or channelized flow over the dam.
- The replacement dam does not rely on the existing dam for support or seepage control. The existing dam can remain unaltered, thus lowering risk of pool loss during construction.
- The replacement dam can be designed to meet modern stability criteria. The dam could be designed (i.e., overbuilt, at a greater cost) to accommodate future crest raises that would allow additional water storage.
- The overflow weir provides an erosion resistant, redundant system to maintain the controlling elevation of the mill race.

Disadvantages of this option include:

- This suite has lengthier, more complex and more expensive design and construction phases. Longer construction duration exposes both the contractor and the owner to potential risks for a longer period of time. The more complex approach may reduce the number of capable marine contractors, when compared to Suite Nos. 1 and 2. However, construction of Suite No. 3 is still within the capabilities of average size marine contractors with average size marine equipment.
- Dredging will generate a larger volume of material (demolition debris, derrick stone, sediment) that must be disposed of at an offsite, upland disposal area. This activity will be a significant contributor to the overall cost.
- The replacement dam will require more complex tie-in features at the rock shelf contact and east abutment. Additionally, bank protection features (e.g., Training Wall Cell) at the east abutment will have to be constructed.
- Requires more extensive bedrock preparation, particularly at any sections that key into sloping or vertical rock faces. Underwater (i.e., diver) work is typically necessary to sufficiently clean cell foundations prior to concreting.

- The replacement dam requires demolition and excavation at the east abutment. During construction, this could temporarily increase the risk of a flanking failure in this area.
- Pinning both the upper and lower lock gates in the open position creates a preferential pathway for water flowing over the new dam, particularly in the near term prior to the eventual failure or breach of the existing timber crib dam. This could lead to scour at the inside toe of both the land and river lock walls.
- This is the only suite which involves activity (including water diversion) in the mill race. Thus, this option carries increased effort and cost (design and construction) for this area.

6.3. Construction Cost Opinion

For purposes of estimating contractor overhead, as well as escalation, a start date of May 2015 and a construction duration of 16 months (2, 8-month seasons) was assumed. The assumed start date was provided by LRL, and the construction duration was assumed based on historical experience with similar projects in Kentucky. Per direction from LRL, a contracting mechanism similar to a Federal Multiple Award Task Order Contract (MATOC) was assumed, with a prime contractor acting as an administrator with limited duties, and a subcontractor who performs the construction work. Note that different assumptions regarding construction duration, start date, and/or contracting mechanism could significantly affect the cost opinion.

The construction cost opinion (i.e., project cost) for this remedial option is approximately \$21,500,000. The cost opinion derivation (from MCACES Second Generation, or MII) can be found in Appendix D. Electronic versions of the native MII files are included in Appendix D on the enclosed CD. In terms of cost, the most significant features of the work include the concrete infill and sheet piling. Significant uncertainties with this option, with respect to cost (i.e., subject to change during future detailed design and/or during construction), are related to the size of the cells required for the new dam. The size of the cells affects sheet piling, concrete, and excavation (and disposal) quantities. The size of the cells is directly related to the depth to rock and the shear strength along the foundation.

As discussed in Section 6.1.1, the rockline is uncertain in certain areas, such as near the rock shelf (where it slopes steeply) and near the east abutment (where borings show rather large variance between borings). Further, portions of the bedrock at the site were found to be soft and/or weathered (Appendix G); thus, the refusal depth for the piling is more uncertain than if the rock were consistently much harder than the overlying soils. Site-specific shear strength parameters have not yet been established (typically based on laboratory testing of large diameter rock core samples). The shear strength of the foundation rock can strongly affect the required cell diameter. As mentioned for Remedial Suite No. 1, transportation costs for hauling of spoil (i.e., dredged material) to an offsite, upland disposal site could be a significant percentage of the total cost. For cost estimating purposes, a haul distance was assumed, but an actual disposal site was not identified.

7. No Action Alternative

Within this scope of work, another alternative to repairing the facility is to take no action. The existing structures would remain in caretaker status or would be transferred to another owner through the disposition process without any modification or repair.

Given that no documented major maintenance has been performed on the dam since 1966, and to the lock since 1977, the structures appear (visually) to be in fair condition. Recent visual observations of the structures during periods of relatively low flow have not revealed any signs of severe distress or indicators of imminent failure (USACE 2007; Stantec 2010b). However, it is evident that the derrick stone placed in 1966 has been reworked in certain areas, creating preferential/channelized flow paths around and beneath stones. Further, it has not been possible to observe or otherwise characterize the interior of the timber crib dam, where voids might exist due to loss of rock fill from within the timber crib frame. Potentially vulnerable components of the structures have been identified, such as the lock gates and exposed areas of the timber crib frame of the dam. Without proper maintenance, the lock and dam cannot be expected to protect the pool adequately for an indefinite period of time.

With respect to potential loss of pool, the following failure modes were identified (Stantec 2010c) and were considered when developing remedial design alternatives: (1) Loss of Dam due to Downstream Scour, (2) Deterioration or Collapse of Dam Crest, (3) Failure of Lock Gates, (4) Erosion of Mill Race Area, (5) Sliding or Rotational Failure of Lock Wall, and (6) Failure of Dam due to Earthquake Loading. If no action were taken to remediate the structures, it is more likely (compared to implementation of Remedial Suite Nos. 1, 2, or 3) that one or more of these failure modes could cause a loss of pool, although the timing of such an event cannot be predicted.

If no action were taken, an indicator of possible future events at Lock and Dam No. 3 is the May 24, 1965 washout (failure) of Lock and Dam No. 4. Although historical documentation is limited, Dam No. 4 suffered a breach of the center portion of the timber crib dam (Figure 14). As shown in the photos, this structure did not have any derrick stone armoring. This event, as well as subsequent “possible failure due to loss of stone from the timber cribs” (Oliver 1987) led to the placement of derrick stone armoring in 1966. The failure of Dam No. 4 may have resulted from deterioration or loss of timber planking/framing, followed by progressive loss of rock fill and further collapse of timber framing. The presence of derrick stone at Dam No. 3 could help to slow or arrest this type of progressive failure by falling into the breach and providing some degree of stabilization. The result might be a narrower breach and/or a slower progressive failure. However, it is also possible that after the initial breach forms, the derrick stone does not shift significantly, and an area of rock fill remains exposed for an extended period, allowing for a larger breach similar to Dam No. 4.



LOUISVILLE DISTRICT, CORPS OF ENGINEERS, U. S. ARMY. Date: 1 June 1965 File No. 37463
 Green River, Locks and Dam #4. General view of dam from Lock River Wall showing condition of upstream decking and crib.



LOUISVILLE DISTRICT, CORPS OF ENGINEERS, U. S. ARMY. Date: 1 June 1965 File No. 37472
 Green River, Locks and Dam #4. General view upstream showing breach in dam.

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Top: Dam 4 – Jun1965_002.jpg
 USACE-LRL

Bottom: Dam 4 – Jun1965_011.jpg
 – USACE-LRL

Figure 14. Post-failure photos of Green River Dam No. 4

As noted above, the lock gates have been noted as vulnerable with respect to failure and loss of pool (Failure Mode 3 as mentioned above). Failure of a gate could happen suddenly, and could result in loss of pool down to the upper sill elevation (7.3 feet below the crest elevation of the dam). Currently, the lock chamber and upper lock approaches are filled with sediment, near or even above the crest elevation of the dam. Given that this area is on the inside of the river bend, and was likely excavated from the river bank during original construction, it is not unexpected that this area tends to fill back in over time. However, future high pool events could generate erosive forces that remove sediment, leaving the highly deteriorated gates as the only means of pool protection in the lock chamber.

If the “no action” alternative was selected, and the pool is lost through a structural failure of the lock, dam, or mill race/rock shelf, several upstream impacts would be realized. In the short term, various bank failures along the Green River and/or Mud River may result due to rapid drawdown. Such landslides are common along major rivers when navigation dams suffer a gate failure that allows the pool to drop rapidly. Buildings, roads, or other infrastructure located immediately next to the river could be affected. In the long term, water intakes in the upper pool could be rendered inoperable, depending on the intake elevation.

With respect to public safety at the project site, the existing risks associated with the lock and access to the site would remain unchanged. Pedestrians could fall from the lock walls and may be unable to exit the chamber in the event of an injury.

Regarding eventual disposition of the property, leaving the property in its current condition may make it more difficult to find a party willing to accept structures that are in need of major repairs. Further, if the structures were to fail prior to disposition, this may make it even more difficult to transfer ownership.

8. Summary and Conclusions

Lock and Dam No. 3 on the Green River is located 108.5 river miles upstream from the Ohio River, in Rochester, Kentucky. Built in the period 1833 to 1838 to enhance commercial navigation, the facility consists of a fixed-crest overflow dam (rock-filled timber crib structure covered with derrick stone) and a stone masonry navigation lock. The dam abuts a shallow rock outcrop (“rock shelf”) that also serves as part of the fixed-crest overflow control for the upper pool. In 1848, a mill was constructed on the rock bluff at the left abutment, and a portion of the rock outcrop was excavated to create a “mill race” that channeled water to power the mill. The lock has not been operated since 1981, when the project was placed in “caretaker status”.

Stantec was charged with the task of performing an assessment of the long term stability and integrity of Lock and Dam No. 3 and developing viable remedial options to a 30% design level with preliminary cost opinions. Major tasks included: historical document review, site geological assessment, geotechnical investigation, laboratory testing, underwater (i.e., diving) assessment and hydrographic survey, stability analyses and integrity assessment, development of preliminary remedial alternatives, and 30% design and cost opinions for three remedial options.

With the assistance of LRL, an extensive historical document review was performed. USACE archives were the main source of information, although several publications on the history of the Green River and other similar topics were also reviewed. Despite the relatively small

amount of historical information available for a facility that has existed since the 1830s, an approximate chronology of site history was constructed for the facility. Numerous repairs, many undocumented, have been undertaken over the years. In most cases, no drawings or other documentation were found for such repairs, making it difficult to ascertain the dates and extent of modifications to various structures.

Subsurface explorations were accomplished by Stantec personnel and equipment during the period of September 29 to November 6, 2009. Fourteen borings were drilled across the site, and can be broken down as follows: two land borings to the west of the mill race (adjacent to Kentucky Highway 70), five land borings to the east of the lock chamber and the upper guide wall, and seven floating plant borings within the Green River, upstream of the dam. A variety of drilling methods were employed to obtain suitable soil and rock samples. Soil samples were subjected to physical (classification) and chemical laboratory testing.

Underwater observations were performed by Stantec's dive team on August 3, 2010. The dive team observed the lock chamber, lower lock approach, riverside face of the river lock wall, and areas upstream and downstream of the dam. A hydrographic survey was performed on August 4, 2010 by the dive team and a professional surveyor. Survey shots were also taken on the exposed portion of the rock shelf, adjacent to the mill race, portions of the derrick stone face, and portions of the left river bank (upstream and downstream).

In general, the alluvial river valley soils fit typical expectations as one proceeds from the outside to the inside of a river meander. The left (west) abutment is on the outside bend of the river and has thin soil deposits overlying shallow bedrock, or rock outcrops without soil cover. In the river channel, soils are thin or not present upstream of the mill race and rock shelf. Moving across the channel towards the right abutment, the alluvium became deeper. Soil in the river is predominantly silty sand, with smaller amounts of clays and gravel. The right (east) abutment is on the inside bend of the river, and has deeper soils. Soils at the east abutment are predominantly silty sand, clays and silts, and smaller amounts of gravel.

The rock found across the site consists of shales and sandstones, fitting the typical rock types expected in the upper portion of the Tradewater Formation. Significant zones of weathered sandstone or soft, friable sandstone were identified in many of the borings. Bedrock is very shallow (little to no soil cover) at the west abutment, as well as in the mill race and rock shelf areas. As would be expected, the rockline drops significantly between the rock shelf area and the area upstream of the timber crib dam. The rockline upstream of the timber crib dam is slightly variable, but does not show a consistent trend in elevation from left to right. The rockline at the east abutment appears to rise several feet from upstream to downstream and also may fall several feet from left to right, which is unexpected, given the location on the inside of the river meander.

Previous studies by USACE and others have indicated the potential presence of PCBs in sediment in this portion of the Green River. The likely source of any PCB contaminated soils in this area is a former industrial site on a tributary of the Mud River, approximately 65 miles upstream of the project site. Per the scope of work, LRL requested that Stantec obtain river sediment samples to test for PCBs, as well as various metals. Other than arsenic, all constituents tested at non-detectable levels, or at levels that were 1 to 4 orders of magnitude (i.e., 10 to 10,000 times) below the "residential soil" Preliminary Remediation Goals (PRGs). The arsenic results are considered within normal background levels for Kentucky soils.

Underwater observations revealed little to no sediment immediately downstream of the derrick stone armoring on the dam. Along the upstream side of the dam, a 25-foot wide zone of tree debris was present in front of the exposed derrick stone, preventing underwater observations in this area. Upstream of this tree debris, a wedge of stone and/or cobble has been placed or has accumulated parallel to the dam. The upper lock approach was inaccessible due to heavy accumulation of sediment and tree debris.

On the exposed face of the dam, derrick stone generally ranged from 4 to 8 feet in diameter. In many areas, highly weathered timbers with metal spikes could be observed between or beneath the stones. This is consistent with the assumption that only one layer of derrick stone was placed over the dam in 1966. Some areas had multiple layers of stone, possibly the result of stones being shifted and redeposited during flood events.

A qualitative failure mode analysis was performed for the various components, to identify specific vulnerabilities to address during design of remedial options. Stantec developed preliminary remedial design alternatives for the rock-filled timber crib dam, masonry lock, and the rock shelf/mill race area with consideration given to maintaining crest elevation, ability to maintain the current pool, public safety, economics, and constructability. During a meeting in January 2011, Stantec presented these preliminary design alternatives to LRL for their consideration. In a February 2011 letter, LRL outlined the three remedial suites that were selected to advance to 30% designs and construction cost opinions.

Remedial Suite No. 1 addresses the rock-filled timber crib dam and the masonry lock but does not address the mill race and rock shelf area. In select areas, derrick stone on the face of the rock-filled timber crib dam will be replenished, back to the approximate grade at which it was installed. In addition, the new derrick stone will be slush grouted in place. The upper lock gates will be buttressed with derrick stone and the lower gates will be pinned open to facilitate egress and to limit accumulation of sediment in the chamber.

Remedial Suite No. 2 addresses the rock-filled timber crib dam and the masonry lock but does not address the mill race area and rock shelf area. To create a uniform crest elevation, sheet piles will be driven to rock upstream of the existing rock-filled timber crib dam and a reinforced concrete cap will be constructed on the upstream sloping portion of the existing dam. The top elevation of the sheet piling and the concrete cap will be equal to the crest of the existing dam. The downstream face of the existing dam will receive treatment identical to Remedial Suite No. 1. As for the lock chamber, a reinforced concrete bulkhead wall and splash pad will be constructed on the upper sill and keyed into the lock walls. The upper and lower lock gates will be pinned back into their recesses to deter pedestrians from accessing the lock chamber and to limit accumulation of sediment in the chamber.

Remedial Suite No. 3 addresses the rock-filled timber crib dam, the masonry lock, and the mill race area. To create a uniform crest elevation, a new cellular concrete dam will be constructed upstream of the existing timber crib dam. The west end of the new dam will tie into the rock shelf. The new dam will extend across the upper lock approach, which eliminates the need for remediation of the upper gates or the lock chamber. A new abutment will be constructed at the east bank, upstream of the existing esplanade. In the mill race area, a concrete overflow weir will be constructed along the same alignment as the new cellular dam. The crest of the weir would be equal to the controlling elevation of the entry to the mill race, which is assumed to be a few feet lower than the crest elevation of the dam.

Within this scope of work, another alternative to repairing the facility is to take no action. The existing structures would remain in caretaker status or would be transferred to another owner through the disposition process without any modification or repair. Potentially vulnerable components of the structures have been identified, such as the lock gates and exposed areas of the timber crib frame of the dam. Without proper maintenance, the lock and dam cannot be expected to protect the pool adequately for an indefinite period of time. If no action were taken to remediate the structures, it is more likely (compared to implementation of Remedial Suite Nos. 1, 2, or 3) that one or more of the identified failure modes could cause a loss of pool, although the timing of such an event cannot be predicted.

Each remedial suite was developed to a 30% design level and the corresponding drawings can be found in Appendix C. In addition, the project cost for each option was estimated using the MII program. Based on the assumptions made herein, Remedial Suite Nos. 1, 2, and 3 have estimated project costs of approximately \$790,000, \$3,300,000, and \$21,500,000, respectively. The detailed MII cost breakdown for each remedial suite is attached in Appendix D. The increasing costs from Suite No. 1 through Suite No. 3 are consistent with the increasing project duration, complexity, and reliability of the remedial options.

While all three of the selected remedial suites are viable alternatives to improve the facility, each has advantages and disadvantages, as well as uncertainties and risks. As future planning and design activities progress, the issues discussed herein, as well as the underlying assumptions should be reviewed and adjustments made if needed.

The nature of the 30% designs includes various assumptions and uncertainties, due in part to the limited data available at this stage of the project. Aspects of one or more remedial suites can be quite sensitive to certain assumptions. One example, discussed in Section 6.1.1, is the shear strength parameters assigned to the foundation of the replacement dam in Remedial Suite No. 3. No laboratory testing data were available, so a friction angle of 30 degrees was assumed based on typical, published values for the rock types observed. The width of the dam is sensitive to this parameter. In future design phases, if the friction angle were found to be 25 degrees (for example), this would result in an increase of roughly 15% in the cross sectional area of each cell. This would translate into significantly higher costs for items such as dredging (and related disposal), sheet piling, and concrete.

Another example of uncertainty is the condition of the existing timber crib dam, which affects Remedial Suite Nos. 1 and 2. Most of the timber crib structure is concealed by derrick stone or by sediment, and the structure cannot easily be explored or otherwise characterized. The ability to successfully treat the existing structure, such that it will perform reliably into the future, cannot be predicted with confidence. Further, construction efforts related to such structures carry additional risk of concealed conditions and associated change orders.

Uncertainty remains regarding the existing conditions at the site, due to incomplete or missing historical documentation, concealed portions of the structures, and the limited nature of the site exploration performed to date. Additional information could influence the probable failure modes discussed herein, as well as the suggested remedial options and costs. For example, there is no information available regarding the presence or condition of filling culverts in the lock walls. If such features exist and have not been properly abandoned, they could represent a potential failure mode that has not been assessed. In general, as new information becomes available, the potential impacts on the assumptions, findings, designs, and costs in this document should be considered.

9. References

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Appendix A

Scope of Work

Appendix B

Selection of Remedial Suites

Appendix C

30% Design Drawings

Appendix D

30% Cost Opinions

Appendix E

Design Calculations

Appendix F

ITR and QA Documentation

Appendix G

Updated Preliminary Findings Report

Appendix H

Stability and Failure Mode
Analysis of Existing
Conditions and
Preliminary Remedial
Design Alternatives